

# Practice of Partial Prestressing for Continuous Post-Tensioned Structures in North America



**Clifford L. Freyermuth**

Executive Director  
Post-Tensioning Institute  
Phoenix, Arizona

The first national recommendations for prestressed concrete in the United States were published in 1958.<sup>1</sup> These recommendations permitted service load tensile stresses of  $6\sqrt{f'_c}$  psi ( $0.5\sqrt{f'_c}$  MPa) with the further provision that this tensile stress level "... may be exceeded provided it is shown by tests that the structure will behave properly under service conditions and meet any necessary requirement for cracking load or temporary overload."

This general code policy remained in effect for both buildings and bridges until 1971 when the ACI 318 Building Code<sup>2</sup> increased the (hypothetical) allowable tensile stress level to  $12\sqrt{f'_c}$  psi ( $1.0\sqrt{f'_c}$  MPa) where "... computations

based on the transformed cracked sections and on bilinear moment deflection relationships show that immediate and long-term deflections comply with code limits." As in previous editions of the ACI 318 Building Code, the 1971 edition provided that the arbitrary tensile stress limits "... may be exceeded when it is shown experimentally or analytically that performance will not be impaired."

Thus, from a historical perspective, American building code provisions have permitted partial prestressing if the concept is defined on the basis of allowing tensile stresses in the concrete under service loads. Unfortunately, the potential economic and serviceability advantages of partial prestressing have not been fully realized in construction practice in North America due to inherent engineering conservatism, lack of technical understanding of the behavior

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Presents an overview of current American design practice in the application of partial prestressing for post-tensioned building and bridge construction. After reviewing the evolution and shortcomings of some provisions in building codes, the author suggests technological areas for future research and code development.

of structures with tensile stresses at service loads, and the lack of a consensus among experts in the field on the technical merits of partially prestressed concrete.

The latter point has contributed to a widespread impression among engineers, particularly in the bridge field, that partial prestressing inherently provides structures of a lower quality than "full" prestressing. For whatever reason, post-tensioned bridge designs in many states are still based on zero allowable tensile stress under full service load. Nevertheless, there has been more progress in the use of tensile stress levels up to  $9\sqrt{f'_c}$  and  $12\sqrt{f'_c}$  ( $0.75\sqrt{f'_c}$  and  $1.0\sqrt{f'_c}$  MPa) in the design of post-tensioned buildings.

### **Influence of Unbonded Tendons on Building Design Technology**

The use of unbonded tendons in buildings was initiated in the United States by T. Y. Lin in 1956-57 with the construction of post-tensioned flat plate roofs for some schools in Nevada. Flat plate construction continued to grow in the 1960's. Typically, the floor and roof slabs were of uniform thickness, were prestressed with unbonded tendons, and contained only minimal amounts of nonprestressed bonded reinforcement.

The average prestress was generally at least 300 psi (2.1 MPa).

Because the tendons were unbonded, concerns arose regarding the structural safety of such systems in the event of multiple anchorage failures. To prevent such an occurrence, the 1971 ACI Building Code introduced provisions for minimum amounts of bonded nonprestressed reinforcement for structures using unbonded tendons. Stated simply, the 1971 ACI requirements provided bonded reinforcement with enough strength to support the dead load plus a small portion of the live load. This reinforcement was (and still is) based on  $0.004A$ , where  $A$  is the area of that part of cross section between flexural tension face and center of gravity of gross section (sq in.).

To utilize this relatively large amount of bonded reinforcement economically, designs under the 1971 ACI Building Code began to use lower prestress levels and higher tensile stress levels under service loads. Consequently, service load tensile stresses in the range of  $6\sqrt{f'_c}$  psi ( $0.5\sqrt{f'_c}$  MPa) became necessary to effectively use the code specified minimum amount of bonded reinforcement in one-way post-tensioned slab systems using unbonded tendons.

For designs where durability considerations are not a factor, service load

Table 1. Recommended span-to-depth ratios for reinforced and prestressed concrete structures in North America.

Structure type	Reinforced concrete	Prestressed concrete
Flat plate building	1/32.73*	1/45 to 1/48†
One-way slab building	1/20 to 1/28*	1/45 to 1/50†
Beams or joists in building	1/16 to 1/21*	1/30 to 1/40†
Multispan box girder bridges	1/18‡	1/25 to 1/30§

\*CRSI Design Handbook.<sup>2</sup>

†Post-Tensioning Manual.<sup>4</sup>

‡Ontario Highway Bridge Design Code.<sup>3</sup>

§Post-Tensioned Box Girder Bridge Manual.<sup>6</sup>

tensile stress levels for one-way post-tensioned systems of  $6\sqrt{f'_c}$  to  $12\sqrt{f'_c}$  psi ( $0.5\sqrt{f'_c}$  to  $1.0\sqrt{f'_c}$  MPa) are now routine. The related average prestress levels range from 180 to 100 psi (1.2 to 0.7 MPa), and even less.

### Restraint Cracking

The majority of significant cracking observed on continuous post-tensioned buildings over the past 25 years resulted from inadequate design and construction considerations of the restraints imposed on the floor system by the supporting columns and/or walls. Flexural cracking of the type normally attributed to partial prestressing has not been significant. Concern about service load flexural crack widths within the hypothetical tensile stress limit [ $12\sqrt{f'_c}$  psi ( $1.0\sqrt{f'_c}$  MPa)], now in the ACI Building Code, does not appear to be warranted.

In conclusion then, the use of partial prestressing, especially as applied to flat slabs, has reduced the incidence of significant cracking problems in post-tensioned buildings. This results from lower prestress levels, inducing proportionately smaller elastic and creep shortening movements, and the presence of bonded nonprestressed reinforcement to control crack widths.

### Economically Viable Span-to-Depth Ratios for Partially Prestressed Structures

The economic viability of prestressed concrete buildings and bridges is directly related to the span-to-depth ratios listed in Table 1. It is apparent that there is a sharp discontinuity between the depths appropriate for conventionally reinforced concrete as compared to prestressed concrete. Since the economy of prestressed concrete structures is directly related to the use of shallow structure depths, there is generally no economic motivation to consider applying prestressing to structure depths appropriate for reinforced concrete. There is obviously a lower level of prestress (partial prestressing ratio or degree of prestress) at which deflection and crack width for shallow span-to-depth ratios used for prestressed structures will not be acceptable.

From a practical standpoint, prestressing below this level is inappropriate for most prestressed concrete applications. For this reason, there is little motivation in developing a unified design approach covering reinforced, partially prestressed, and fully prestressed concrete structures. Rather, efforts should be concentrated in developing appropriate criteria for designing shal-

low prestressed concrete structures with sufficient amounts of prestress to ensure satisfactory serviceability.

In summary, the acceptable and economically viable structural depths of reinforced concrete and prestressed concrete structures are distinctly different. It is apparent that the slab thickness used for a given prestressed slab would prove to be unserviceable and uneconomical if used as conventionally reinforced. From the perspective of the prestressed concrete industry there is interest only in designs with combinations of prestressed and nonprestressed reinforcement for which the shallow span-to-depth ratios normally used are satisfactory. The current ACI Building Code provisions permit degrees of prestress that cover the practical spectrum for most prestressed concrete applications.

### Need for Simplicity

As pointed out by Bachmann,<sup>7</sup> partially prestressed concrete will not be fully realized in practice in North America unless the design method is simple and easy to apply. It might be relevant to mention that one major factor in the acceptance of post-tensioned building construction in the United States was the introduction of the load balancing design method by Lin<sup>8</sup> in the early 1960's. And yet initially, there was much criticism of the load balancing method because several minor approximations are assumed in the design procedure. Experience has shown, however, that these approximations do not have any detrimental effect on the behavior of the structure.

With regard to partially prestressed concrete, requirements such as time consuming and complicated calculations of prestress losses and deflection; precise evaluation of stresses at service load in nonprestressed and prestressed reinforcement; and detailed computation of crack widths would all be con-

sidered detrimental to the acceptance of partial prestressing in North America.

Nevertheless, it would appear that such requirements could be simplified or omitted to realize most of the economic and serviceability advantages of partially prestressed structures.

## BUILDING DESIGN

Nearly all post-tensioned building construction in the United States is based on the use of unbonded tendons in conjunction with code specified minimum amounts of bonded nonprestressed reinforcement. Maximum design tensile stresses range from  $6\sqrt{f'_c}$  to  $12\sqrt{f'_c}$  psi ( $0.5\sqrt{f'_c}$  to  $1.0\sqrt{f'_c}$  MPa). Span-to-depth ratios are generally comparable to those indicated in Table 1 for prestressed concrete.

### One-Way Slabs

Table 2 presents information on comparative designs of four three-span one-way slabs. Dimensions and other details of the slabs are shown in Fig. 1. Slabs A and B in Table 2 are the prototype designs for slabs tested by Burns et al.<sup>9</sup> at the University of Texas at Austin with design tensile stresses of  $6\sqrt{f'_c}$  and  $9\sqrt{f'_c}$  psi ( $0.5\sqrt{f'_c}$  and  $0.75\sqrt{f'_c}$  MPa), respectively.

Slab C is designed for a maximum tensile stress (hypothetical) of  $12\sqrt{f'_c}$  psi ( $1.0\sqrt{f'_c}$  MPa) and Slab D was designed for zero tensile stress under service load. The span-to-depth ratio for all slabs is 43.64. The results of a rigorous computer analysis of Slabs A, B, C, and D are given in the Appendix.

### Slabs A and B

The half-scale model tests of Slabs A and B indicated satisfactory strength and serviceability characteristics in both cases. The first visible crack in Slab B, designed for a service load tensile stress

Table 2. Comparative design of one-way continuous post-tensioned slabs with various design tensile stress levels.

Slab designation	Design tensile stress $\sqrt{f'_c}$	Design tensile stress, psi	$P/A$ stress, psi	$A_{pv}$ in <sup>2</sup> /ft	ACI Code $A_s$ Eq. (18-6) in. <sup>2</sup> /ft	$A_s$ provided, in. <sup>2</sup> /ft	Percent redistribution, ACI Sect. 18.10.4.1	$M_2$ , in.-kips	$1.7D + 1.4L$ (max) $M_u$	Design moment, in.-kips	Moment capacity $M_{u,ps}$ , in.-kips	Total moment capacity provided, in.-kips	PPR $\frac{M_{u,ps}}{M_u}$	Moment capacity $\frac{M_{u,ps}}{\text{Design moment}}$	Degree of prestress $K$	Percent of end span dead load balanced
A	$6\sqrt{f'_c}$	379	185	0.086	0.132	0.092	13.4	4.91	93.98	76.48	68.7	92.00	0.747	0.898	0.61	78
B	$9\sqrt{f'_c}$	569	140	0.062	0.132	0.15	13.6	3.60	93.98	77.60	49.7	87.90	0.57	0.640	0.45	57
C	$12\sqrt{f'_c}$	759	85	0.039	0.132	0.187	15.4	2.26	93.98	77.25	29.4	79.33	0.37	0.381	0.28	36
D	0	0	300	0.138	0.132	0.132	9.6	7.97	93.98	76.99	108.5	138.2	0.79	1.409	1.00	126

Metric (SI) conversion factors:

 $6\sqrt{f'_c}$  psi =  $0.5\sqrt{f'_c}$  MPa;  $9\sqrt{f'_c}$  psi =  $0.75\sqrt{f'_c}$  MPa;  $12\sqrt{f'_c}$  psi =  $1.0\sqrt{f'_c}$  MPa.1 ft = 0.305 m; 1 in. = 25.4 mm; 1 in.<sup>2</sup> = 645.2 mm<sup>2</sup>; 1 psi = 0.006895 MPa; 1 in.-kip = 113 N.m.

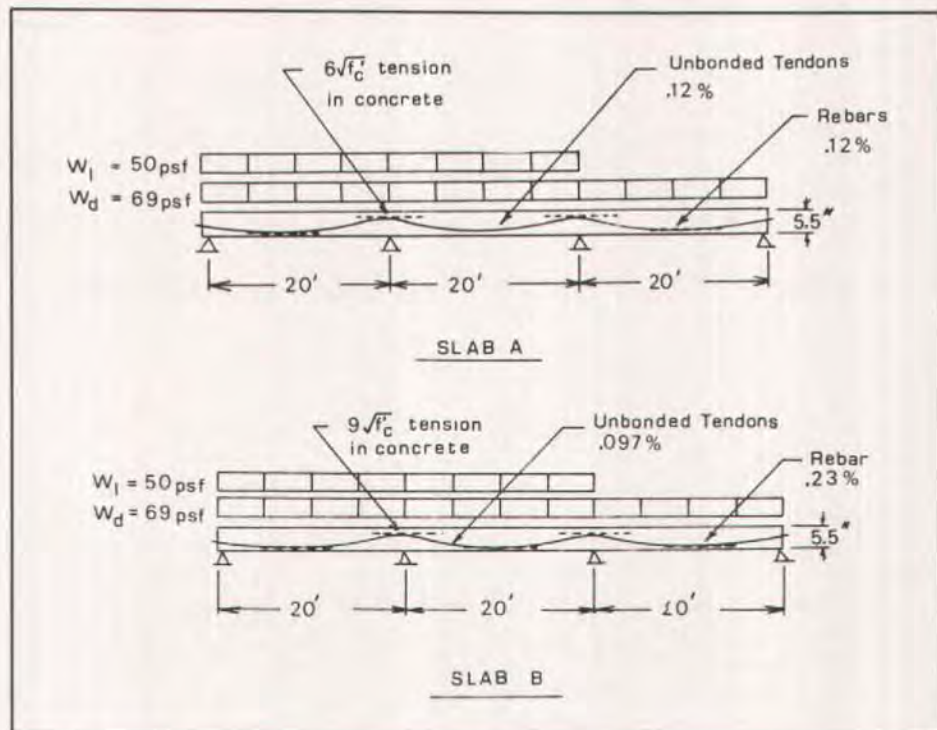


Fig. 1. Prototype design parameters for Slabs A and B.

of  $9\sqrt{f'_c}$  psi ( $0.75\sqrt{f'_c}$  MPa), occurred at a load level of  $1.0D + 1.6L$ . However, the load-deflection curve for Slab B indicated that cracking did begin at about the service load level ( $1.0D + 1.0L$ ). Of course, the fact that the tests indicated no cracking at service load level for Slab B, designed with a maximum tensile stress of  $9\sqrt{f'_c}$ , means that considerations of flexural crack width are superfluous for such designs.

The maximum service load deflection measured in the test of Slab B was  $L/569$  as compared to allowable value of  $L/360$  specified in the ACI Building Code. However, the tests did indicate that the calculated deflection, considering a bilinear load-deflection curve, was unconservative for evaluating the deflection behavior of a slab which is initially cracked. The maximum calculated service load deflection for Slab B was 0.162

in. (4 mm) as compared to the measured value of 0.211 in. (5 mm).

This indicates the need to develop improved procedures for calculating deflections for continuous slabs designed with tensile stresses of  $9\sqrt{f'_c}$  to  $12\sqrt{f'_c}$ . Nonetheless, the deflection behavior of the slab was acceptable, and it might be inferred that the code deflection limits are intrinsically satisfied by slabs designed with tensile stresses of  $9\sqrt{f'_c}$  and span-to-depth ratios comparable to those in Table 1, without requiring deflection calculations.

Due to the 3 to 1 ratio between the cost of prestressing steel and reinforcing steel, Slab B, with less prestressing steel and more nonprestressed reinforcement would be a slightly more economical design than Slab A.

This shows that for continuous post-tensioned slabs the most economical

combination of reinforcing bars and prestressing steel is not necessarily associated with the minimum total area of steel. In this case, a degree of prestress of about 0.6 (Slab A) does indicate the design with minimum steel area, but not necessarily the most economical solution.

### Slabs C and D

Slab C was included in Table 2 to show that the existing code limit of  $12\sqrt{f'_c}$  psi ( $1.0\sqrt{f'_c}$  MPa) on the hypothetical tensile stress is probably near the practical limit for one-way slabs. The amount of prestressing provided is relatively small.

The computer analysis of Slab C (see Appendix) indicates a maximum service load deflection of 0.963 in. (24 mm), or  $L/250$ , which exceeds the Code limit of  $L/360$ . This suggests that the ACI Code limit of  $12\sqrt{f'_c}$  psi ( $1.0\sqrt{f'_c}$  MPa) on the hypothetical tensile stress encompasses the serviceability range for span-to-depth ratios normally used.

The design for Slab D indicates the wastefulness of full prestressing for this application. The tendons alone provide 140 percent of the strength required, and the tendons plus minimum code reinforcing bars provide 180 percent of the strength requirement. Since the prestress balances 126 percent of the dead load, some camber growth might be anticipated.

The cost of reinforcement for Slab D would be significantly greater than for the other three designs and the serviceability behavior may be less satisfactory, particularly from the standpoint of possible cracking due to increased restraint to elastic and creep shortening by supporting columns and walls. Note that the shortening of Slab D due to prestress would be more than 200 percent greater than Slab B.

Finally, it might be noted that the partial prestressing ratio (PPR) is not particularly meaningful for Slab D since

the slab is fully prestressed and still has a PPR of 0.79. This would be true for any fully prestressed slab with unbonded tendons designed under the ACI Building Code since the Code requires a minimum amount of bonded reinforcement when unbonded tendons are used.

### Load Balancing Design Criteria

Most of the criteria which have been proposed to measure the level of prestress have been expressed as a function of the reinforcement or stresses at midspan of a simple span beam. While these criteria (PPR and degree of prestress) can be applied to continuous structures in many cases, these factors relate to only one section of the structure at a time, and they are, therefore, somewhat limited with respect to providing an index of the expected behavior of the structure as a whole.

On the other hand, the load balancing concept (introduced and developed by T. Y. Lin<sup>8</sup> in the early 1960's), can be a useful index as to the expected behavior of the deflection characteristics of a structure. The method is particularly valuable in the design of prestressed flat slabs. As indicated by the last column in Table 2, it can be inferred that one-way slab designs for which 50 to 80 percent of the dead load is balanced (Slabs A and B) can be expected to satisfy the ACI Code serviceability requirements and be more economical alternatives for reinforcing the slab.

Furthermore, a higher or lower amount of dead load might be balanced depending upon the amount of deflection or cracking considered acceptable for the given situation. With regard to Slabs A and B for which 78 and 57 percent of the dead load was balanced, respectively, it is again emphasized that no cracking was visible on the model slabs until loaded well above the service load level.

Lastly (and most importantly), load

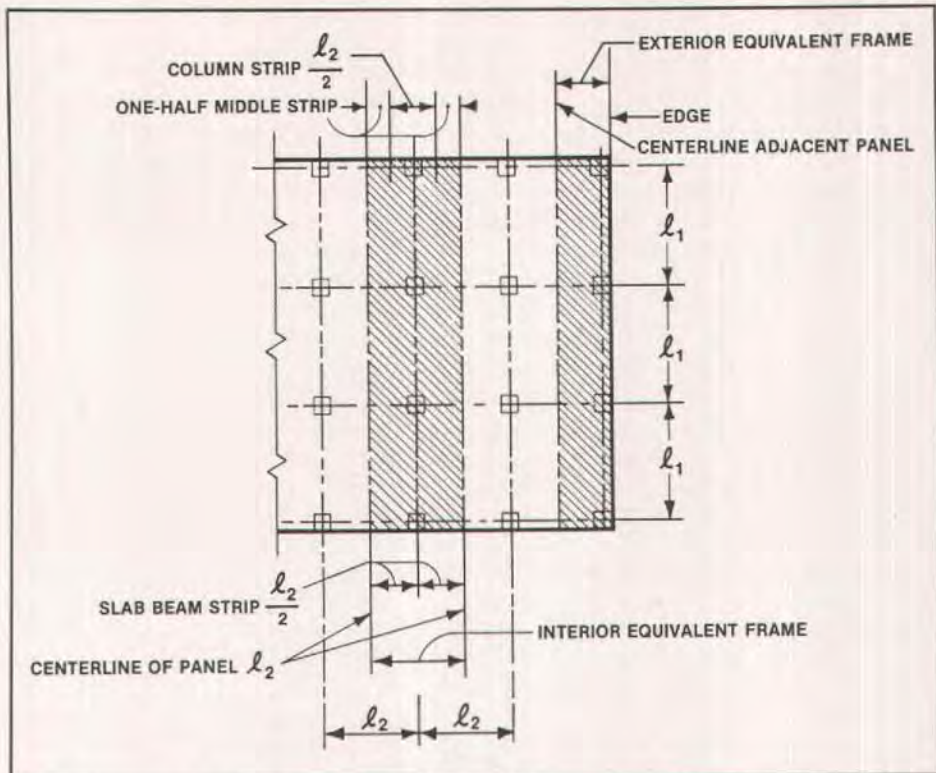


Fig. 2. Equivalent frame definition for design of two-way flat plates.<sup>2</sup>

balancing provides an easy way to estimate the amount of prestressing steel required and a simple procedure for calculating secondary (hyperstatic) moments due to prestressing.

### Flat Plates

Under the ACI Building Code, flat plates (defined as slabs of uniform thickness reinforced in two directions) are designed as wide flat beams with the beam width extending from center to center of adjacent panels, as shown in Fig. 2.

Fig. 3 (see Ref. 10) compares stresses in a flat plate at the centerline of a column as determined by beam theory (the ACI Code approach), plate theory, and by experimental strains in a four panel flat plate. Note that the beam theory

moments at the center of the column are only one-third the value of the moments determined by experimental strains.

At other locations in the flat plate, the beam theory moments provide a good approximation of the experimental moments. However, in recognition of the gross approximation of the actual strains in the immediate vicinity of the column, the ACI Building Code limits the stress at that location as calculated by the beam theory to  $6\sqrt{f'_c}$  psi ( $0.5\sqrt{f'_c}$  MPa), as compared to the maximum value of  $12\sqrt{f'_c}$  psi ( $1.0\sqrt{f'_c}$  MPa) for one-way slabs.

Several laboratory tests of large models of post-tensioned flat plates<sup>11</sup> have shown that proportioning of reinforcement on the basis of beam theory provides structures meeting the strength and serviceability criteria of the ACI

Building Code. This observation also is sustained by the satisfactory behavior of about 400 million sq ft (37 million m<sup>2</sup>) of post-tensioned flat plate floor and roof construction that has been completed to date in the United States.

With regard to partial prestressing, it would appear that the extension of flat plate designs to higher allowable tensile stress values would require the use of a finite element analysis or other more sophisticated analytical procedures to evaluate stresses in the immediate vicinity of the columns.

While analytical procedures (together with computer programs) are now available<sup>12</sup> to evaluate the response of a structure throughout its elastic, cracking, inelastic and ultimate ranges, application of these procedures on a routine basis is presently beyond the capability of most designers, and the cost of such

analyses is probably not warranted in most design cases.

As in the case of one-way slabs, the ACI Building Code requires the use of bonded reinforcement in flat plates prestressed with unbonded tendons, particularly in the immediate column vicinity. This reinforcement contributes to both the flexural and shear strength at the column. Because of the presence of bonded reinforcement and the normal service load tensile stress limit of  $6\sqrt{f'_c}$  psi ( $0.5\sqrt{f'_c}$  MPa), flat plate designs in the United States have been partially prestressed since the advent of the 1971 ACI Code.

Also, since average prestress levels in flat plates as low as 125 psi (0.9 MPa) are common under the current ACI design parameters, the potential economic motivation for using higher allowable stress levels and more precise analytical pro-

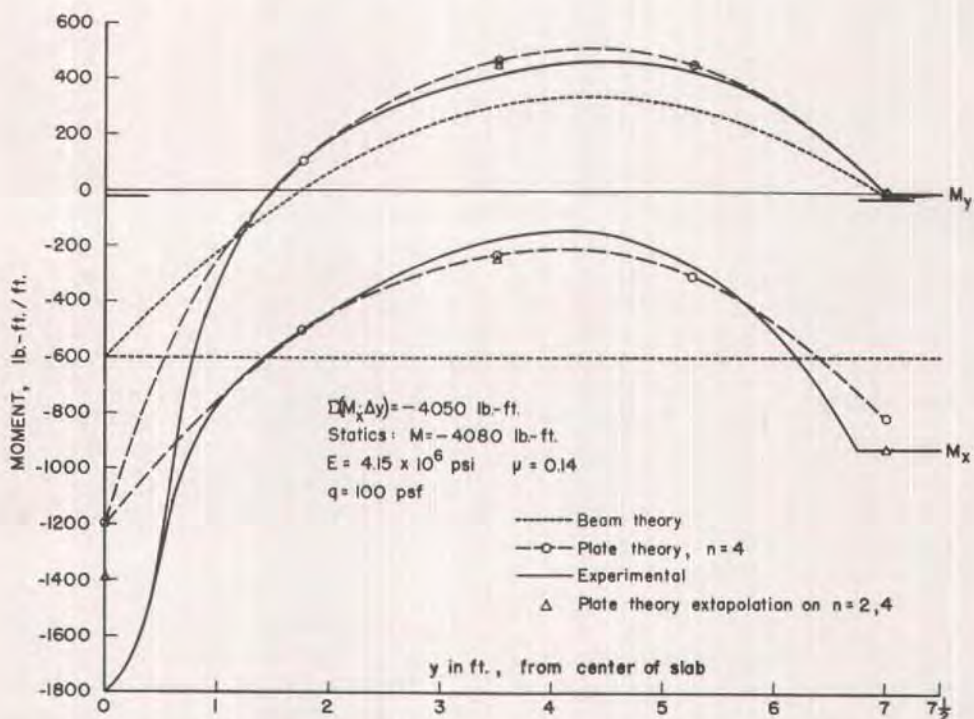


Fig. 3. Comparison of beam theory, plate theory, and experimental moments in post-tensioned flat plate at center of column.<sup>10</sup>

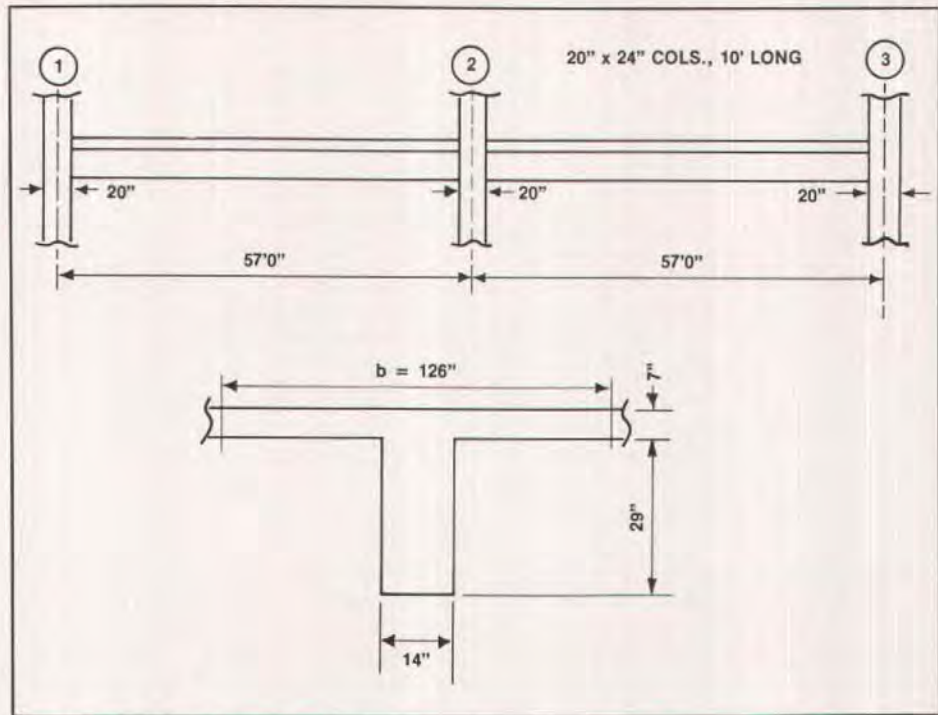


Fig. 4. Two-span T beam parking structure.<sup>4</sup>

cedures is limited. Nevertheless, higher allowable stress levels are permitted in conjunction with more precise analytical procedures following Section 18.4.3 of ACI 318-83.

### T Beams

Fig. 4 shows a two-span T beam for a parking structure taken from the Post-Tensioning Manual.<sup>4</sup> The 36 in. (914 mm) beam depth used in this example is about 6 in. (152 mm) deeper than normally used. The design allowable tensile stress is  $6\sqrt{f'_c}$  psi ( $0.5\sqrt{f'_c}$  MPa).

On the basis of this allowable tensile stress, the required post-tensioning forces at various locations are as follows:

- Columns 1 and 3 ..... 52.05 kips
  - Midspan ..... 348.78 kips
  - Column 2 ..... 281.29 kips
- (Note: 1 kip = 4.448 kN)

Considering the arbitrary  $6\sqrt{f'_c}$  psi

( $0.5\sqrt{f'_c}$  MPa) tensile stress limit, the design was based on the 348.78 kips (1,551 kN) force required at midspan. A selective, allowable tensile stress increase to  $12\sqrt{f'_c}$  psi ( $1.0\sqrt{f'_c}$  MPa) at the midspan location (tensile stresses of this magnitude at the bottom of the beam are more acceptable from a durability standpoint) would provide a 19 percent reduction in the required post-tensioning force [to 281.29 kips (1,251 kN)] at the cost of a relatively small amount of additional bonded reinforcement at midspan and at the interior column.

The secondary or hyperstatic moments, due to post-tensioning based on the 348.78 kip (1,551 kN) tendon force, reduced the design (ultimate) moments by 37 percent at Columns 1 and 3, 24 percent at Column 2, and increased the design moment by 42 percent at midspan. Accordingly, it is apparent that the secondary moments are a significant

factor in the design of this T-beam for strength.

It is believed that the American practice of including secondary moments in strength calculations with a load factor of 1.0 has been adequately substantiated by experimental and analytical studies.<sup>13,14</sup> While significant approximations in stress calculations may be safely permitted, comparable approximations in strength calculations are not acceptable. It is further believed that secondary moments are a function only of the forces induced in the tendon during stressing (adjusted for prestress losses).<sup>13</sup> Load induced increases in tendon strains are statically neutralized or equilibrated by the loads themselves.

Since load induced strains are not considered in continuous, conventionally reinforced concrete beams to generate secondary moments, there appears to be no logical basis for such a practice for prestressed concrete beams. Accordingly, a single load factor of 1.0 for secondary moments is considered appropriate as specified in Section 18.10.3 of the ACI Building Code.

## BRIDGE DESIGN

As noted in the introduction, tensile stresses at service loads of  $6\sqrt{f'_c}$  psi ( $0.5\sqrt{f'_c}$  MPa) are permitted for post-tensioned bridges by the AASHTO Specifications.<sup>15</sup> However, design practice in many states is based on zero allowable tensile stress. This conservatism appears to be based on the erroneous presumption that allowable tensile stresses in the concrete inherently result in cracking and, in some cases, related apprehensions about the influence of cracking on the durability of structures exposed to corrosive environments.

Ironically, if a zero tensile stress criterion were applied to conventionally reinforced concrete bridges, it would mean that such structures could not be built. In the case of prestressed concrete, bridges can be built but at the cost of providing additional prestressing steel and conventional reinforcement.

It is felt that a wider understanding of the techniques and merits of partially prestressed concrete would lead design engineers to use at least the  $6\sqrt{f'_c}$  psi

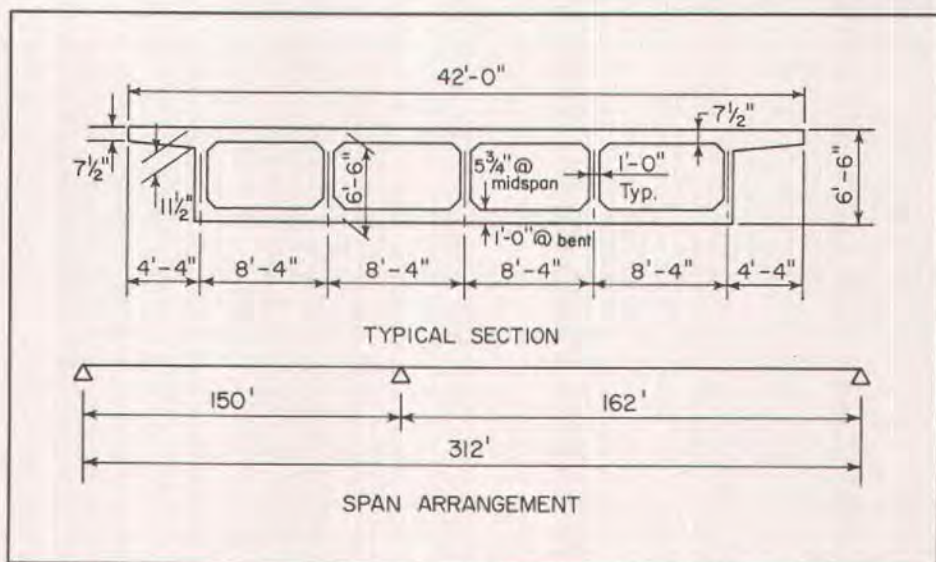


Fig. 5. Typical section and span arrangement for two-span box girder bridge.<sup>6</sup>

Table 3. Comparison of moments and reinforcement requirements for zero and  $6\sqrt{f'_c}$  allowable tensile stress.

Tensile stress, psi	Design moment, ft-kips	1.2 × Cracking moment, ft-kips	Moment capacity of tendons, ft-kips	Conventional reinforcement required, in. <sup>2</sup>
Zero	58,812	61,972	65,547	0
	61,689	50,673	50,309	32

Metric (SI) conversion factors:

1 psi = 0.006895 MPa; 1 ft-kip = 1,356 N · m;

1 in.<sup>2</sup> = 645.2 mm<sup>2</sup>;  $6\sqrt{f'_c}$  psi =  $0.5\sqrt{f'_c}$  MPa.

( $0.5\sqrt{f'_c}$  MPa) allowable tensile stress limitation now included in the AASHTO Specifications.

### Reduction in Tendon Force Requirement by Using $6\sqrt{f'_c}$ Allowable Tensile Stress

The two-span box girder bridge shown in Fig. 5 was designed for an allowable tensile stress of  $6\sqrt{f'_c}$ , or 424 psi (2.9 MPa) for 5,000 psi (34.5 MPa) concrete.<sup>6</sup> The initial tendon jacking force required to satisfy this stress limitation is 7,418 kips (3,301 kN), based on a tensile stress of 424 psi (2.9 MPa) at the top of the section over the interior column.

Revision of the allowable tensile stress to zero at this location requires an increase in the initial tendon jacking force to 9,625 kips (52,731 kN). Accordingly, a 30 percent reduction in the quantity of prestressing steel required is possible in this case by changing the limiting tensile stress from zero to  $6\sqrt{f'_c}$  [424 psi (2.9 MPa)].

Although the design with  $6\sqrt{f'_c}$  tensile stress may require some additional conventional reinforcement at peak moment locations, the economy in total reinforcement cost afforded by the higher allowable tensile stress is significant.

### Cracking Moment Reinforcement as a Function of Allowable Tensile Stress (see Table 3)

Due to possible brittle collapse of grossly under-reinforced sections at the time of cracking, the AASHTO Specifications,<sup>15</sup> Section 1.6.10(B), require reinforcement sufficient to develop a capacity equal to 1.2 times the cracking load, calculated on the basis of the modulus of rupture of  $7.5\sqrt{f'_c}$  psi ( $0.625\sqrt{f'_c}$  MPa). Design for zero tensile stress results in a tendon moment capacity 11 percent greater than the design moment, and a value of 1.2 times the cracking moment which is also greater than the design moment.

The design for  $6\sqrt{f'_c}$  psi ( $0.5\sqrt{f'_c}$  MPa) results in a much more economical mix of reinforcement and a value of 1.2 times the cracking moment that is substantially less than the design moment. For some span arrangements, the 1.2 times cracking moment provision can require substantial amounts of bonded reinforcement which serve no purpose, e.g., unbalanced span arrangements which result in large residual compressive stresses at some sections.

Although the AASHTO Specifications presently do not permit moment redistribution, the reinforcement index for the zero tensile stress design is only 0.074 — much less than the maximum

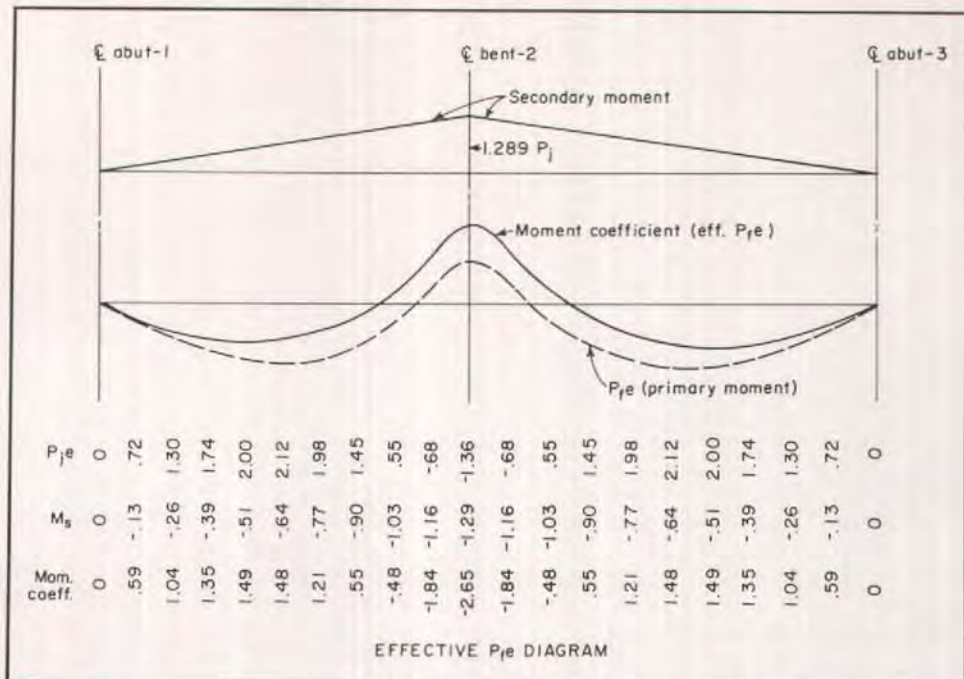


Fig. 6. Primary and secondary moments for two-span box girder bridge.<sup>6</sup>

value of 0.3 which ensures yielding of the steel before failure. In general, application of AASHTO Section 1.6.10(B) to the design of post-tensioned box girder bridges is considered unnecessary and sometimes very wasteful.

### Secondary Moments

Fig. 6 illustrates the primary and secondary moments in the two-span box girder bridge illustrated in Fig. 5 designed for  $6\sqrt{f'_c}$  tensile stress. Note that the secondary moments are 95 percent of the primary moment at the center support and 25 percent of the primary moment at the point of maximum positive moment. The secondary moments reduce the design (ultimate) moment at the center support by 14 percent, and increase the design moment at the point of maximum positive moment by 10 percent.

Accordingly, the secondary moments

play a significant role at both the serviceability and strength levels for this bridge, and for continuous post-tensioned bridges in general.

### Fatigue

Tests on cracked, simple span, precast prestressed girders stressed in fatigue loading to  $6\sqrt{f'_c}$  at the Portland Cement Association<sup>16</sup> and the University of Texas at Austin (unpublished) indicate possible fatigue problems for such members. The tests at the University of Texas indicate that fatigue life can be increased dramatically (in excess of 10 million cycles) by use of properly detailed and tied bonded reinforcement, in addition to prestressing tendons.

Conventional reinforcement is normally used at the point of maximum moment in post-tensioned bridges in the United States. However, the correct detailing procedures to ensure adequate

fatigue resistance will have to be developed for all types of prestressed concrete bridges before design tensile stresses in excess of  $6\sqrt{f'_c}$  can be used.

Flexural cracking would not normally be expected in highway bridges at the  $6\sqrt{f'_c}$  design tensile stress level due to conservative AASHTO Specifications for live load and live load distribution. Of course, it has been demonstrated that the fatigue life of uncracked prestressed flexural elements will extend indefinitely.<sup>17</sup>

The significance of fatigue considerations in bridge designs diminishes as both the span and the dead load to total load ratio increase. The need for fatigue considerations can be evaluated by comparing the stress range in the prestressing tendons and conventional reinforcement to the known fatigue characteristics of prestressing steel and conventional reinforcement.<sup>18</sup>

## FUTURE CODE DEVELOPMENT

A major research need for both fully and partially prestressed buildings is in the area of seismic design. A comprehensive state-of-the-art report on seismic resistance of prestressed and precast concrete structures has been presented which identifies the research needs for building structures.<sup>19</sup>

Shear design provisions for prestressed concrete in the ACI Building Code are believed to become more conservative as the degree of prestress decreases. However, in some cases these provisions may be excessively conservative for partially prestressed members and development of more unified shear design criteria seems appropriate.

With reference to the one-way slab tests at the University of Texas at Austin, the bilinear deflection calculation procedure currently in the ACI Building Code is unconservative for previously cracked, continuous prestressed mem-



Fig. 7. 400 East Ohio Street Apartment Building, Chicago, Illinois.

bers. Use of higher design tensile stress levels for partially prestressed continuous members will require development of more precise methods for calculating deflections.

Design and construction provisions to ensure the durability of structures exposed to aggressive environments will be a major thrust of the 1989 ACI Building Code. Such considerations will obviously be of significance for the design of partially prestressed structures with very low prestress levels.

As discussed earlier, fatigue design is currently a major obstacle to using higher tensile stress levels (greater than  $6\sqrt{f'_c}$ ) associated with partially prestressed concrete design for bridges.

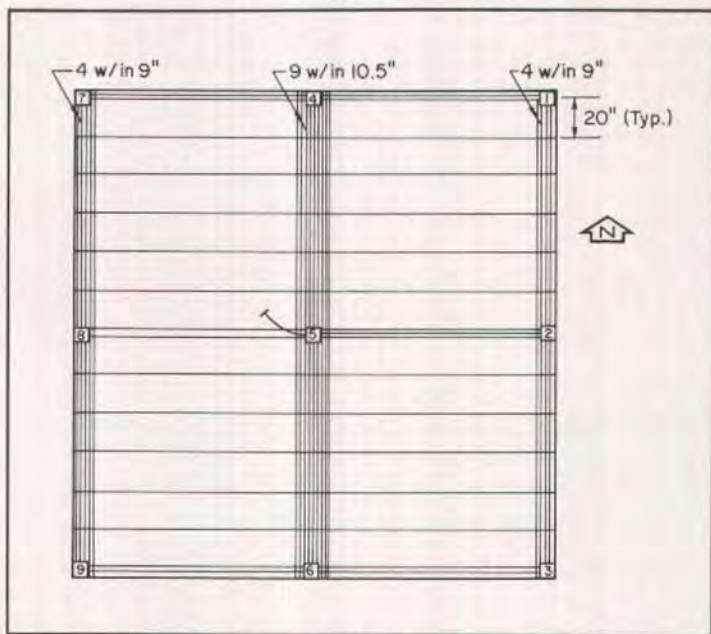


Fig. 8. Flat plate banded tendon layout.

## APPLICATIONS

The 400 East Ohio Street Building, shown in Fig. 7, is a 50-story apartment building located north of the Chicago Loop. The building has four service floors below 46 apartment floors, and an adjacent six-story garage.

The 7 in. (178 mm) thick one-way tower slabs were designed using a combination of post-tensioning tendons in the long direction and conventional reinforcement in the short direction of the building. The post-tensioning tendons were used as primary reinforcement in the two end bays with 25 ft 10 in. (7.9 m) spans, and as temperature steel for the interior bay.

Similarly, at the two end bays of the building, the mild steel in the transverse direction functioned as temperature steel, and for the 22 ft (6.7 m) span interior bays the conventional transverse steel made up the main reinforcement. The maximum service load tensile stress for the post-tensioned one-way slabs

was  $6\sqrt{f'_c}$ . The 7 in. (178 mm) thick floor system (maximum span-to-depth ratio 44.3) was used as the minimum for incorporating the mechanical system within the slab thickness.

The East Ohio Building provides an interesting combination of reinforced and prestressed concrete design in a one-way slab floor system.

Most flat plate construction in the United States presently uses banded tendon distribution as shown in Fig. 8. This method of tendon installation greatly simplifies the construction process. Fig. 9 shows the Key Colony Condominium in Key Biscayne, Florida which incorporates about 1 million sq ft (0.093 million  $m^2$ ) of post-tensioned flat plate construction. The maximum design tensile stress was  $6\sqrt{f'_c}$ .

Fig. 10 shows the 35-story Parklane Plaza condominium in Houston, Texas. This residential structure is a banded tendon building with a combination of one-way and two-way floor system designs. The 18 ft (5.5 m) cantilever at the



Fig. 9. Key Colony Condominium, Key Biscayne, Florida.

corners of the building is supported by tendons and by conventional reinforcement, as shown in Fig. 11.

The use of post-tensioned cast-in-place box girder bridges, such as those shown in Figs. 12 and 13, represents a major portion of the bridge construction market throughout the United States. As noted previously, some states design on the basis of  $6\sqrt{f'_c}$  tensile stress whereas others design for zero tensile stress.

The California bridges shown in Figs. 12 and 13 were designed for a tensile stress of  $6\sqrt{f'_c}$ .

## CONCLUSION

This paper is intended to provide an overview of current American practice in the application of partial prestressing for post-tensioned building and bridge construction in addition to suggesting needs for future research and code development.

In building construction, the use of



Fig. 10. Parklane Plaza Condominium, Houston, Texas.

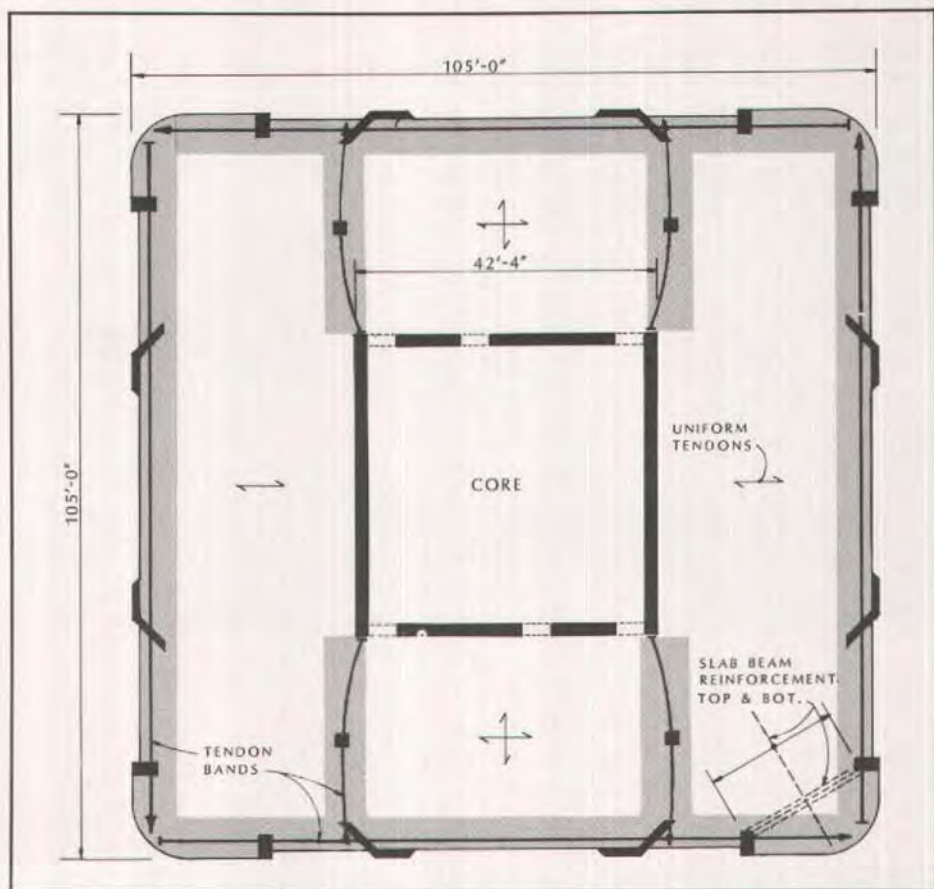


Fig. 11. Typical structural floor plan (Parklane Plaza Condominium, Houston, Texas).

partial prestressing is well advanced, due largely to the widespread use of unbonded tendons in conjunction with code specified amounts of bonded conventional reinforcement. The economical span-to-depth ratios for prestressed building elements imposes a lower limit on the amount of prestress consistent with serviceability (deflection) limitations.

The viability of designs with apparent tensile stress levels in excess of  $12\sqrt{f'_c}$  seems questionable. It appears unnecessary to calculate crack width, deflection, or tensile stresses in reinforcement for building designs with tensile stress less than  $9\sqrt{f'_c}$ . It is suggested that such

calculations be required only where tensile stresses exceed  $9\sqrt{f'_c}$ .

A load factor of 1.0 is considered appropriate for consideration of secondary (hyperstatic) moments at the design (strength) load level. The use of load balancing is considered to be a useful index of structural behavior as well as a meaningful criterion for evaluating the level of prestress.

There is a need for developing a new technology for the seismic design of prestressed buildings with all levels of prestress. Further development also appears necessary in methods of calculating deflections for continuous, cracked prestressed structures, and in design for

durability of structures exposed to aggressive environments.

Bridge engineers in North America are less inclined to use tensile stresses in design due to concern about cracking and related durability apprehensions. Major economies are possible in reinforcement costs for post-tensioned bridges by use of a modest design tensile stress of  $6\sqrt{f'_c}$ . Such structures should not experience flexural cracking at this stress level (which is well below the modulus of rupture) considering the conservative live load and live load distribution provisions incorporated in the AASHTO Specifications.

Further research is needed to develop detailing recommendations for conventional reinforcement to ensure the fatigue life of cracked bridges with tensile stresses of  $6\sqrt{f'_c}$  or more, particularly for relatively short spans.

Preliminary test results of simple span, cracked precast prestressed I-beams at the University of Texas at Austin, stressed in fatigue loading to  $6\sqrt{f'_c}$ ,

indicate a fatigue life in excess of 10 million cycles when conventional bonded reinforcement is provided and well tied with stirrups.



Fig. 12. Route 274/15 Interchange, San Diego County, California.



Fig. 13. Coyote Creek Bridge, Santa Clara County, California.

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**NOTE:** Discussion of this paper is invited. Please submit your comments to PCI Headquarters by September 1, 1985.

# APPENDIX — STUDY OF CONTINUOUS ONE-WAY SLABS WITH UNBONDED POST-TENSIONING TENDONS AND VARIOUS AMOUNTS OF PRESTRESSING AND MILD STEEL

Prepared by

**Alex C. Scordelis**

Consulting Engineer  
El Cerrito, California

The purpose of this study was to provide a rigorous computer analysis\* of the results obtained in Table 2 in which the author presented a simplified comparative design of four one-way continuous slabs (A, B, C, and D) with various design tensile stress levels.

The initial task was to perform a nonlinear analysis of Slab C (see Table 2) using the computer program "PCFRAME." This program can be used to predict the response of a prestressed concrete frame through its service load history as well as throughout its elastic, inelastic, and ultimate load ranges in one complete analysis.

The theoretical basis and capabilities of PCFRAME are described in a paper by Kang and Scordelis entitled "Non-linear Analysis of Prestressed Concrete Frames," *Journal of ASCE Structural Division*, February 1980, pp. 445-462. (A summary of the capabilities of this program also appears in the Nov.-Dec. 1984 PCI JOURNAL.<sup>12</sup>)

After performing the analysis of Slab C, it was decided to also analyze Slabs A, B, and D in Table 2. Note that Slabs C1 and D1 were identical to Slabs C and D except that they contained no mild

reinforcing steel, i.e.,  $A_s = 0$ .

A summary of the pertinent assumptions and results follows. Additional voluminous data are available in the computer printout.

## Material Properties

A summary of the initial data is given in Table 2 and in the figures. The same material properties were used in all of the analyses, and are summarized below.

Concrete:

$f'_c = 4000$  psi at 28 days

$f_t = 7.5 \sqrt{f'_c} = 474$  psi

$w = 150$  pcf (unit weight)

$\epsilon_{cu} = 0.003$  ultimate compressive strain

Assume the Hognestad stress-strain curve.

Reinforcing steel:

$f_u = 60$  ksi yield stress;  $\epsilon_{su} = 0.16$

Assume an elasto-plastic stress-strain curve.

$E_{s1} = 30 \times 10^6$  psi;  $E_{s2} = 6.25 \times 10^4$  psi



Prestressing steel:

Assume unbonded prestressing strands.

The multilinear stress-strain curve is taken from the curve shown in Fig. A1.

$f_{pu} = 270$  ksi;  $f_{pe} = 144$  ksi

Friction was ignored in all analyses.

\*This study was conducted as a result of a letter from Clifford L. Freyermuth to Professor Alex C. Scordelis dated April 6, 1984, in connection with the paper the author presented at the Advanced Research Workshop on Partial Prestressing, sponsored by NATO Scientific Affairs Division, Paris, France, June 18-22, 1984.

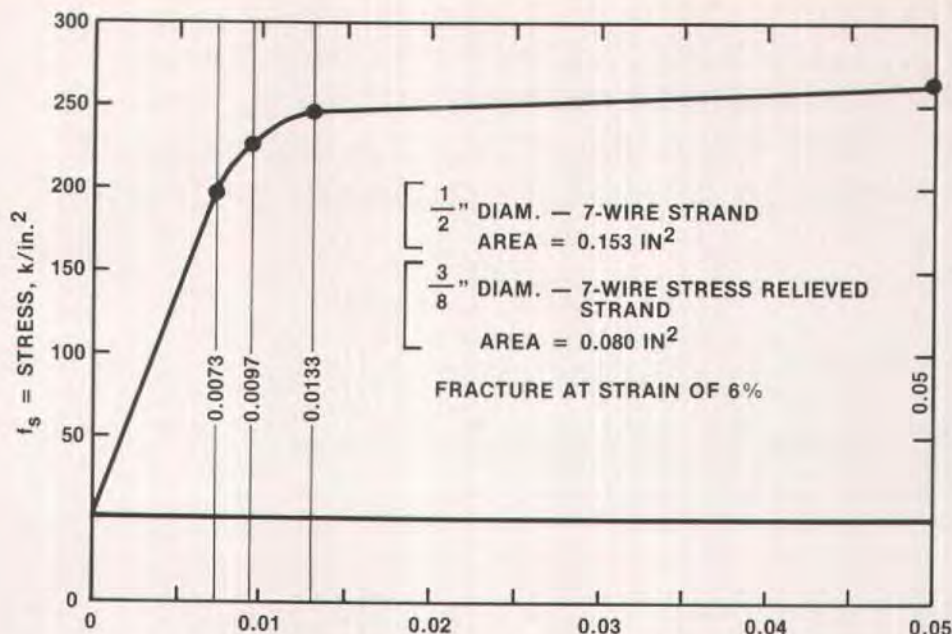


Fig. A1. Stress-strain curve for prestressing steel. Note:  $1 \text{ in.}^2 = 645.2 \text{ mm}^2$ ;  $1 \text{ ksi} = 6.895 \text{ MPa}$ .

## Summary of Results

### FIG. A2

1. The analytical model for PCFRAME consists of 20 nodal points interconnected with 19 elements and supported by stiff springs to measure reactions.

2. The concrete and reinforcing steel are represented by layers at Sections I, II, and III.

3. The same model was used for all analyses, with various values of  $A_s$ ,  $A_{ps}$ ,  $F_e$ , but all with the same material properties.

### FIG. A3

1. The analytical model of post-tensioning tendons consists of 19 straight elements. The geometry ( $x$ ,  $y$  coordinates) was determined from best fit to a smoother parabolic layout.

### TABLE A1 and FIG. A4

1. Loading sequence for Slab C basic

analysis. Incremental loading with one or two loads in each increment was used. Output for displacements, reactions, moments, strains, stresses, etc., were obtained after each load step.

### FIG. A5

1. Load versus displacement for Slab C ( $\delta$  at Node 3; first span).

2. Load versus  $f_s$  in Element 7 for Slab C at first interior support.

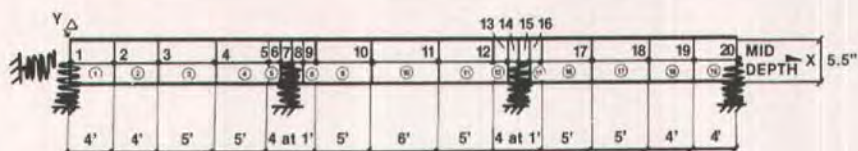
3. Load versus  $f_{ps}$  in Element 7 for Slab C at first interior support.

4. Note that cracking occurs at  $F + DL + 1.0LL$  with a large increase in  $f_s$ .

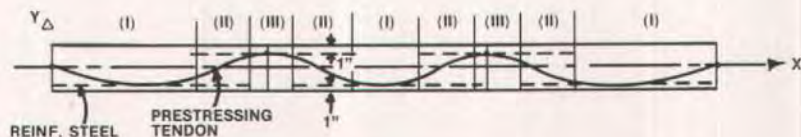
5. For this solution, a maximum of 15 iterations is required. In each load step, substantial unbalanced forces still existed for loads greater than  $F + 1.4 DL + 2.1 LL$ . The steel stress  $f_s$  has already reached 60 ksi at an earlier load.

### FIG. A6

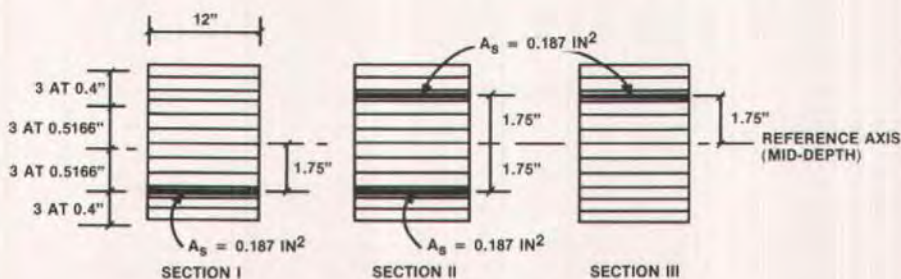
1. Same as Fig. A5 only with a maximum of 200 iterations in each load



(A) STRUCTURAL LAYOUT



(B) PRESTRESSING AND REINFORCING STEEL LAYOUT



(C) CROSS-SECTIONAL LAYOUT

Fig. A2. Analytical model of three-span one-way Slab C. Note: 1 in. = 25.4 mm; 1 in.<sup>2</sup> = 645.2 mm<sup>2</sup>.

step. Results are similar to Fig. A5. Ultimate load occurs at  $F = 1.4DL + 2.3 LL$  with  $\delta = 6.4$  in.,  $f_s = 60$  ksi;  $f_{ps} = 172$  ksi.

TABLE A2

1. Summary of  $A_s$ ,  $A_{ps}$ , and  $F_e$  for analysis of Slabs A, B, C1, C, D1, and D to be subjected to prestress plus increasing uniform load  $w$  (psf) on all spans.

TABLE A3 AND FIG. A7

1. Loading, sequence for analysis of Slabs A, B, C1, C, D1, D. Note that  $F + 0.01$  lb used in first step gives essen-

tially the effect of prestress "F" alone. Subsequent load steps are 10 psf each on all spans.

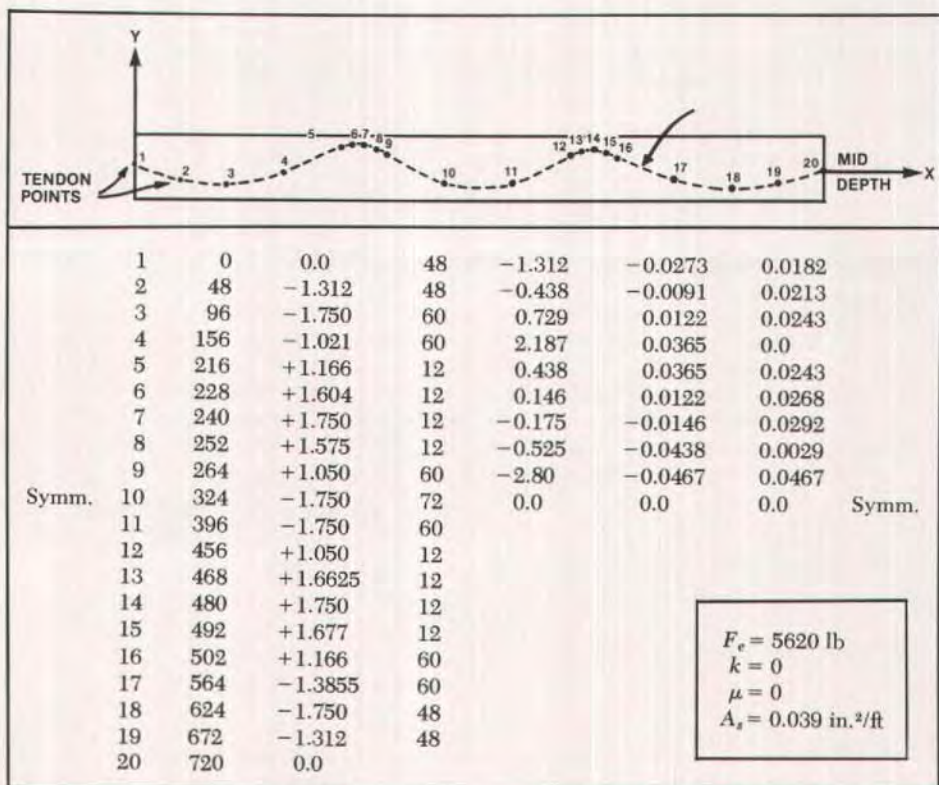
2. Maximum number of iterations in each load step = 50. During these load steps and iterations the nonlinear analysis traces cracking, nonlinear  $\delta$  versus  $\epsilon$ , etc., of concrete, reinforcing steel, and prestress.

FIG. A8

1. Load versus displacement at 0.4 span from exterior support (near the maximum  $\delta$  in first span) for Slabs A, B, C1, C, D1, and D under  $F$  plus uniform load on all spans.

Table A1. Loading sequence for Slab C.

Loading sequence		Number of loading steps
1.	F + 0.5 DL	2
2.	F + 1.0 DL	2
3.	F + 1.0 DL + 0.5 LL	2
4.	F + 1.0 DL + 1.0 LL	2
5.	F + 1.2 DL + 1.35 LL	2
6.	F + 1.4 DL + 1.7 LL	2
7.	F + 1.4 DL + 1.9 LL	1
8.	F + 1.4 DL + 2.1 LL	1
9.	F + 1.4 DL + 2.3 LL	1
10.	F + 1.4 DL + 2.5 LL	1
11.	F + 1.4 DL + 2.7 LL	1
12.	F + 1.4 DL + 2.9 LL	1
13.	F + 1.4 DL + 3.1 LL	1
14.	F + 1.4 DL + 3.3 LL	1
15.	F + 1.4 DL + 3.5 LL	1
16.	F + 1.4 DL + 3.7 LL	1

Fig. A3. Geometry and layout of prestressing tendon (Slab C). Note: 1 lb = 4.448 N; 1 ft = 0.305 m; 1 in.<sup>2</sup> = 645.2 mm<sup>2</sup>.

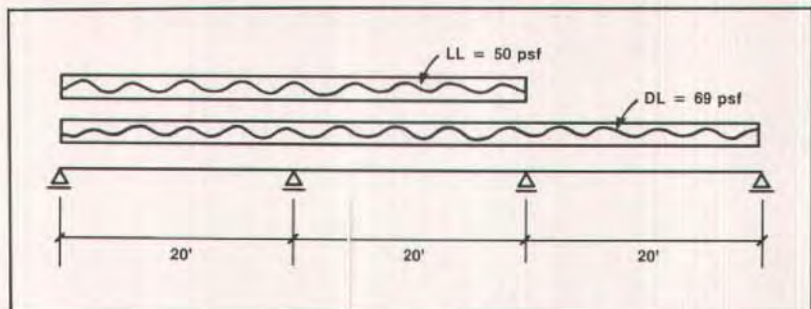


Fig. A4. Loadings for Slab C. Note: 1 ft = 0.305 m; 1 psf = 0.04788 kPa.

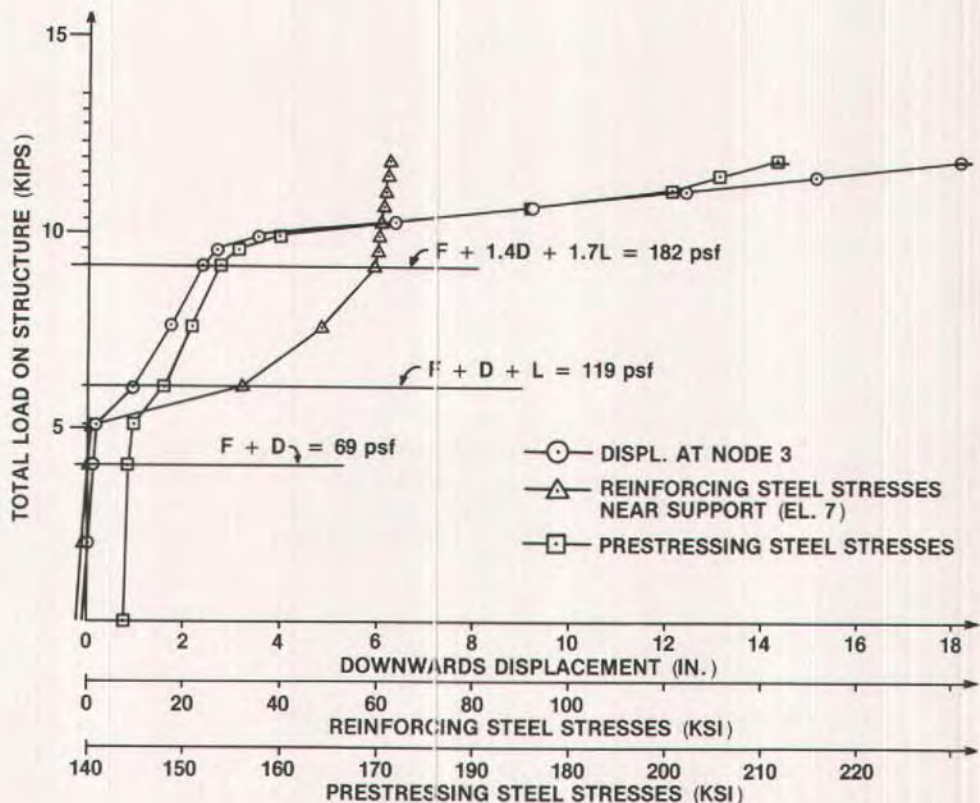


Fig. A5. Results for Slab C with maximum of 15 iterations. Load =  $F + DL + LL$  for maximum negative moment. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa; 1 psf = 0.04788 kPa.

Table A2. Summary of prestressing and reinforcing data for various slabs.

Slab	$A_s$ , in. <sup>2</sup>	$A_{ps}$ , in. <sup>2</sup>	Prestressing force, $F_e$ , lb
A	0.092	0.086	12210
B	0.15	0.062	9240
C1	0.0	0.039	5610
C	0.187	0.039	5610
D1	0.0	0.138	19800
D	0.132	0.138	19800

Metric conversion factors: 1 in.<sup>2</sup> = 645.2 mm<sup>2</sup>; 1 lb = 4.448 N.

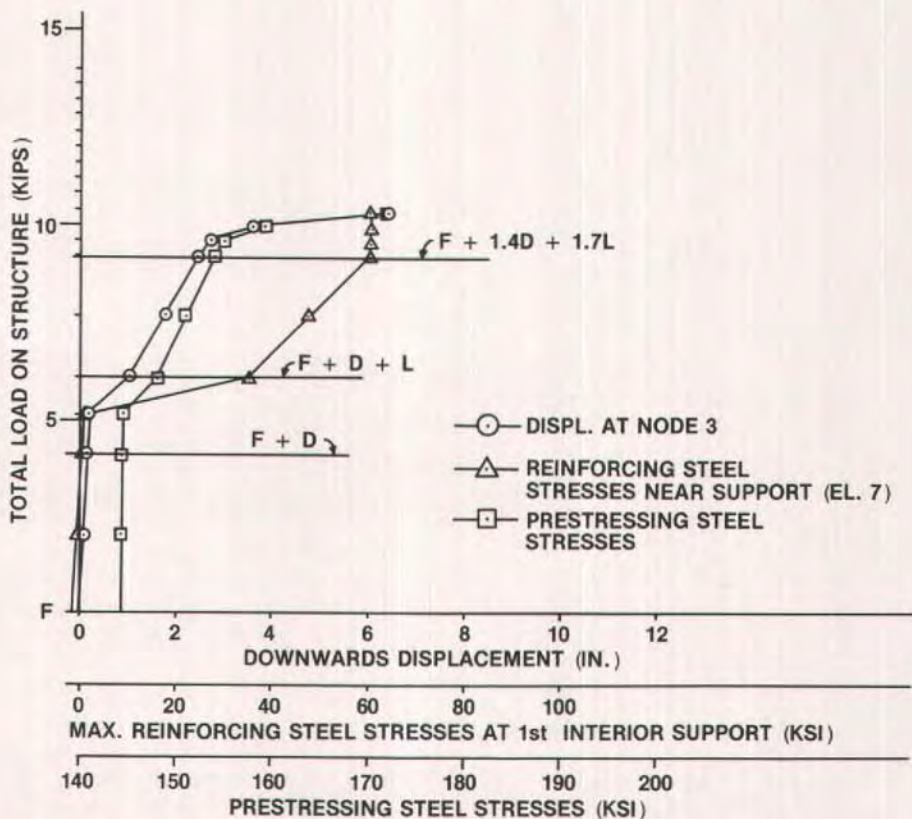


Fig. A6. Results of Slab C with maximum of 200 iterations. Load = F + DL + LL for maximum negative moment. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

Table A3. Loading sequence for all sections (A, B, C1, C, D1, D).

Three time steps with $t = 0$ between them Time Step 1: $F + 0.01$ lb at Node 3 Time Step 2 and 3: $F + 10$ psf for each load step until failure occurs Maximum number of iterations on all load step = 50
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Note: 1 lb = 4.448 N; 1 psf = 0.04788 kPa.

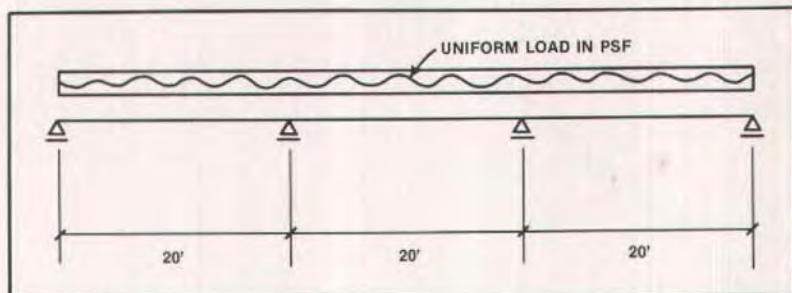


Fig. A7. Loadings for Slabs A, B, C1, C, D1, D. Note: 1 ft = 0.305 m; 1 psf = 0.04788 kPa.

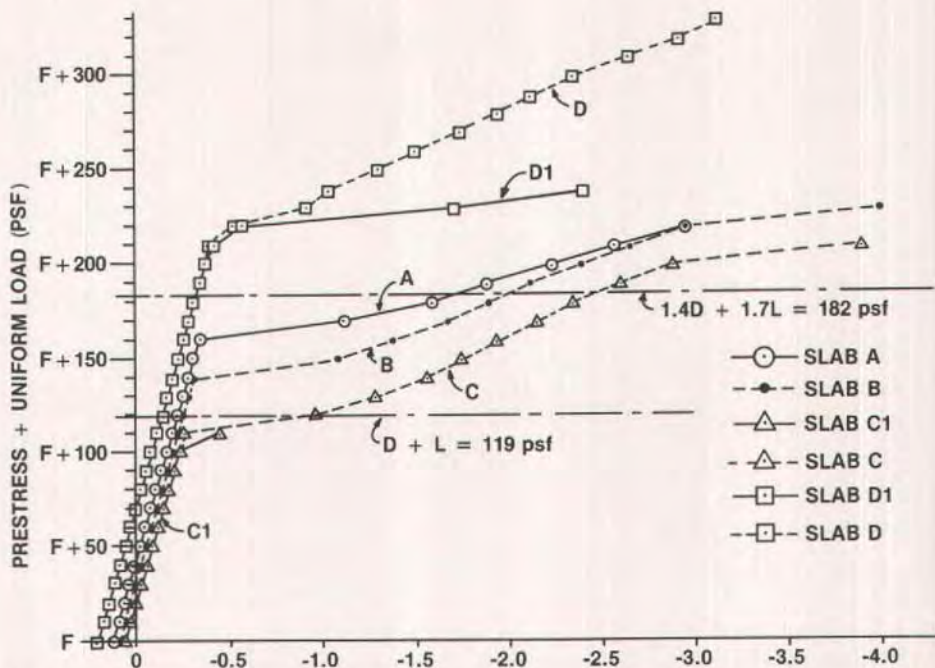


Fig. A8. Vertical displacement at 0.4 span from exterior support (in.). Note: 1 in. = 25.4 mm; 1 psf = 0.04788 kPa.

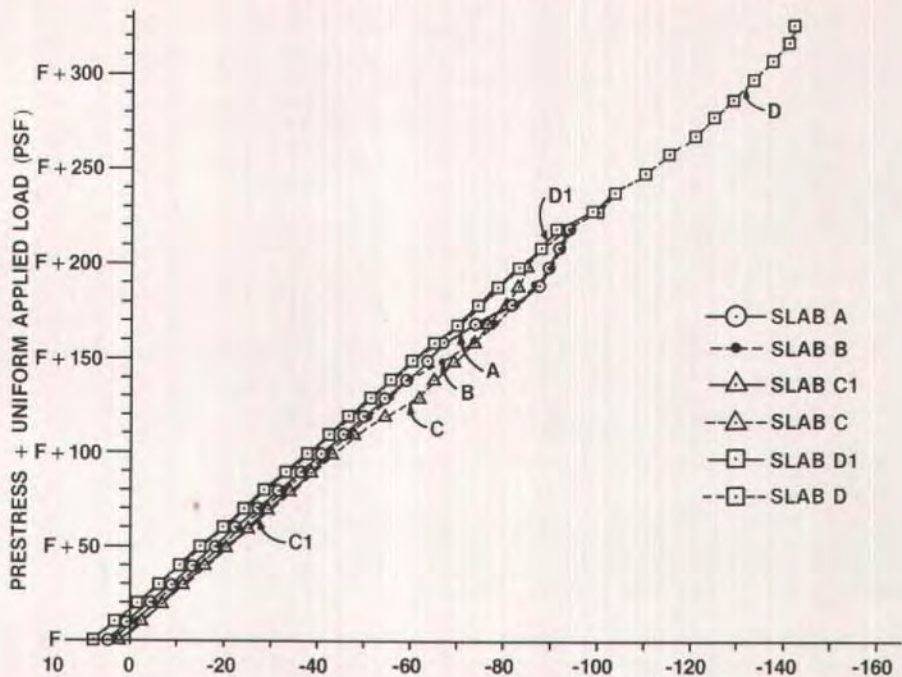


Fig. A9. Moment at interior support (in.-kips). Note: 1 psf = 0.04788 kPa; 1 in.-kip = 113 N • m.

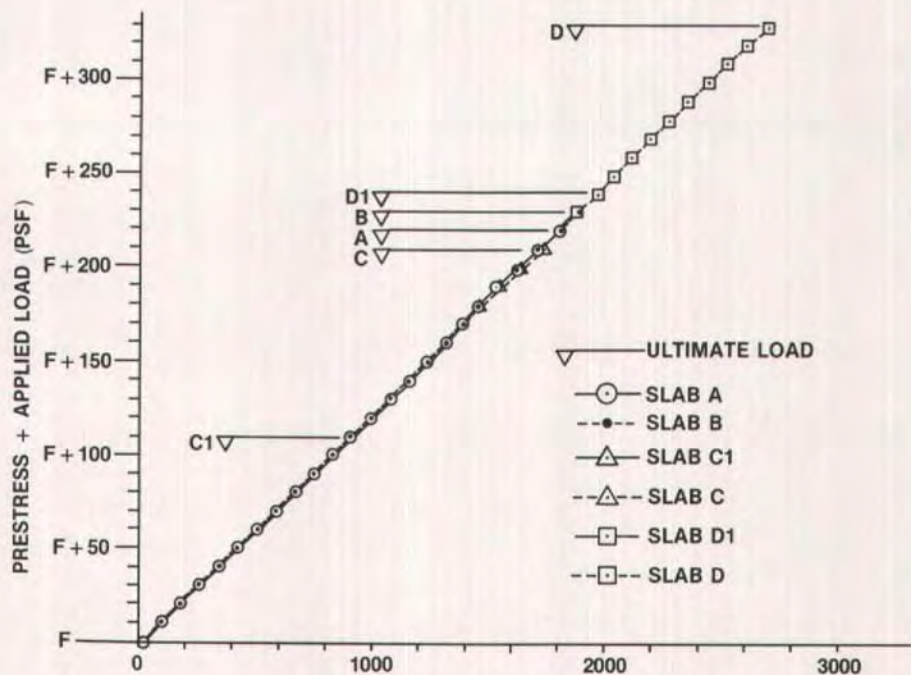


Fig. A10. Reaction at exterior support (lb). Note: 1 psf = 0.04788 kPa; 1 lb = 4.448 N.

Table A4. Summary of prestressing steel stresses and ultimate loads for various post-tensioned slabs.

Slab	A	B	C1	C	D1	D
(1) Initial prestress ( $f_{si}$ )	142.0	149.0	143.8	143.8	143.5	143.5
(2) Prestress at ultimate (ksi)	164.5	180.6	146.8	175.1	161.2	165.8
(3) Ultimate load (psf)	F+220	F+230	F+110	F+210	F+240	F+330
(4) 1983 ACI prestress at ultimate (ksi)	162.2	173.2	173.8	173.8	159.9	159.9
(5) 1977 ACI prestress at ultimate (ksi)	182.6	201.6	203.8	203.8	172.7	172.7

(a) Items (1), (2), and (3) are analytical values found using the PCFRAME computer program. Ultimate load taken at point where deflections continued to increase without convergence of unbalanced load.

(b) Items (4) and (5) are 1983 and 1977 ACI Building Code values for ultimate stress in unbonded post-tensioned tendons for slabs.

$$\text{1983 Code } f_m + f_w + 10,000 + \frac{f_c'}{300 \rho_p} \text{ Eq. (18-5) but not greater than } (f_w + 30,000)$$

$$\text{1977 Code } f_m + f_w + 10,000 + \frac{f_c'}{100 \rho_p} \text{ Eq. (18-4) but not greater than } (f_w + 60,000)$$

Note: 1 ksi = 6.895 MPa; 1 psf = 0.04788 kPa.

2. Ultimate load can be taken at maximum load shown when convergence could no longer be obtained.

3. Note break in linear relation occurs when concrete cracks. Slabs with higher prestress remain linear longer.

4. Slab without mild reinforcing steel C1 and D1 fails sooner than those with it, e.g., Slabs C and D. Substantial "ductility" exists in the slabs with mild reinforcing steel.

#### FIG. A9

1. Load versus moment at first interior support.

2. Note linear relation until cracking. Nonlinearity occurs after cracking due to moment redistribution and possible

change in secondary moments. These effects cannot be separated out easily. However, secondary moments due to this loading are quite small.

3. It should be emphasized that the moments shown and computed by PCFRAME are the section internal moments obtained by integrating the contributions of the concrete and reinforcing bar stresses in layers and the contribution of prestressing stress in the tendon. They thus equal the external moments at the sections due to external loads and reactions.

#### FIG. A10

1. Load versus reaction at exterior support. Essentially a linear relation

exists up to an applied load of 150 psf for Slabs A, B, D, D1, and D (C1 failed at 110 psf). Some slight nonlinearities exist after 150 psf.

2. Note that moment at first interior support is highly sensitive to small changes in exterior reaction since it is due to the difference of two contributions of similar size, namely, those due to the reaction and the applied load.

#### TABLE A4

1. Summary of prestressing steel stresses and ultimate loads for Slabs A, B, C1, C, D1, D under uniform load on all spans.

2. For a dead load:

$$DL = \frac{5.5 (12)}{144} = (150) 69 \text{ psf}$$

and a live load = 50 psf,

$$1.4 DL + 1.7 LL = 182 \text{ psf}$$

Note that all slabs satisfy this criterion, except Slab C1,  $w_u = 110$  psf, which is less than the service load (see note in Item 3 below).

3. Comparing the prestress steel stress at ultimate  $f_{ps}$  by: (2) PCFRAME; (4) 1983 ACI Code; and (5) 1977 ACI Code, it can be seen that (5) is much larger than (2), but (4) agrees quite well with (2) except for Slab C1 where the 1983 ACI Code still overestimates  $f_{ps}$  at ultimate. Note that the Slab C1 design does not comply with the ACI requirements for bonded reinforcement or strength.

Metric conversion factors:

1 psi = 0.006895 MPa;

1 ksi = 6.895 MPa;

1 pcf = 16.02 kg/m<sup>3</sup>; 1 psf = 0.0478 kPa;

1 lb = 4.448 N.

\* \* \*